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DISCUSSION OF MOVEMENTS IN THE DESICCATED ALKALINE SOILS OF BURMA

(*Published in January, 1950*)

By Hans F. Winterkorn, J. E. Jennings, and
F. L. D. Wooltorton

SOIL MECHANICS DIVISION

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DISCUSSION

HANS F. WINTERKORN,[18] Assoc. M. ASCE.—The phenomena observed and reported in this paper, as well as the underlying soil-physical causes, have been of considerable interest to the highway engineer ever since low-cost road construction and soil stabilization began to be transformed from an art into an applied science. Thus far, however, these phases have not been considered to any great extent by the foundation engineer grounded in soil mechanics alone and not in the wider aspects of soil physics.

One of the reasons for this lack of attention on the part of the latter may be that the type of light building that suffers damage as a result of the swelling or shrinking of the underlying soil represents too small an investment to require detailed foundation studies and protective measures. This reason no longer holds, however, if such trouble occurs over a considerable area and if either the government or a private organization is financially involved in a large number of light structures in such an area. For instance, in the drought years of the 1930's a considerable number of light buildings founded on the Putnam soil series in Missouri showed distress fundamentally similar to that described by Mr. Wooltorton. Oil companies owning a large number of service stations in this area suffered appreciable financial loss and were definitely worried about the situation.

The Burma soil under consideration was a calcium-magnesium-sodium solonchatic solonetzig complex. Mr. Wooltorton points out that the high sodium and magnesium content suggested detrimental volume changes and impermeability but that the replaceable calcium and magnesium suggested permeability, and that, although the soils were quite impermeable in laboratory tests, actual moisture movement in the soils on the sites proved them to be rather pervious.

Although the permeability of disturbed soils is definitely a function of the exchangeable ions, it also depends on the way the cations have been introduced into the exchange complex. This is illustrated in Table 5 which gives permeability coefficients as a function of the exchange ions for three different soils, prepared by leaching with chloride solutions of the respective cations (Cols. 2, 3, and 4). For one of these soils (Putnam), Col. 5, Table 5, also shows the effect of neutralization of the H-soil with the respective hydroxides. The Cecil clay is kaolinitic, whereas the Hagerstown and Putnam clays are predominantly illitic, as are the Burma clays reported by Mr. Wooltorton. The ionic influence is less for the clays prepared by leaching than for those prepared by neutralization. A further influence is the salt content of natural soil water

NOTE.—This paper by F. L. D. Wooltorton was published in January, 1950. The numbering of footnotes, tables, and illustrations in this Separate is a continuation of the consecutive numbering used in the original paper.

[18] Industrial and Eng. Consultant, Princeton, N. J.

to which soils are more pervious than they are to the distilled water normally used for laboratory tests.

However, even considering the greater permeability to natural soil water containing electrolytes, it would seem that the observed moisture movements required the action of agencies other than the normally considered ones. An important point in this connection is the increase to a maximum moisture content between 12 ft and 6 ft in May before the rains broke. Mr. Wooltorton (see Section 5) considers, as such an agency, the distillation effect described by A. F. Lebedev.[6,7] Results of studies at Princeton University (Princeton, N. J.) make it very probable, if not certain, that this "distillation" effect is actually a thermo-osmotic film flow, the crucial proof for the latter concept being the electrokinetic phenomena observed as a result of thermo-osmotic flow.

TABLE 5.—Coefficients of Permeability, Computed From Consolidation Experiments on Homo-ionic Soils (Data, Modified by 10^8, Are in Centimeters per Second; Load Range, 1 to 2 Kg per Sq Cm or 14 to 18 Lb per Sq In.)

Ions	Soils Leached with Salts[a]			Neutralization of H-soil[b] (Putnam)
(1)	Cecil (2)	Hagerstown (3)	Putnam (4)	(5)
Natural.....	4.3	4.4	1.0	2.29
H.....	5.4	3.8	3.4	0.75
Na.....	4.8	0.9	1.1	0.056
K.....	5.0	1.5	1.3	2.58
Mg.....	5.5	4.8	3.4	0.60
Ca.....	4.8	2.3	1.5	1.50
Ba.....	4.8	1.8	4.0	....
Al.....	3.8	4.0	13.0	....
Fe.....	5.1	3.6	5.7	....

[a] "Surface Chemical Factors Influencing the Engineering Properties of Soil," by Hans F. Winterkorn, *Proceedings*, Highway Research Board, National Research Council, Vol. 16, 1936, pp. 293–308. [b] "A Study of Changes in Physical Properties of Putnam Soil Induced by Ionic Substitution," by Hans F. Winterkorn, *ibid.*, Vol. 21, 1941, pp. 415–434.

The Princeton experiments permitted the computation of thermo-osmotic permeability coefficients in centimeters per second, per degree centigrade per centimeter, in a manner similar to that of the electro-osmotic permeability coefficients. Typical data reported[19] by the United States Navy in 1949 are given in Table 6 for three pure H-clays of montmorillonitic, illitic, and kaolinitic character, as well as for the natural (illitic) New Jersey Hagerstown soil and six of its homo-ionic modifications. The data show that the observed thermo-osmotic permeability coefficients are of the order of magnitude of 10^{-6} as compared with 10^{-8} to 10^{-12} for hydrostatic coefficients, which means that

[6] "The Movement of Ground Water and Soil Water," by A. F. Lebedev, *Proceedings and Papers*, 1st International Cong. of Soil Science, Vol. 1, 1927.
[7] "Relation of Water to Soil," by A. F. Lebedev, 2d International Cong. of Soil Science, Vol. VI, Comm. VI, Moscow, 1930.
[19] "Report on Beach Sand Stabilization," by K. R. S. Gowda and Hans F. Winterkorn, Bureau of Yards and Docks, U. S. Navy, Washington, D. C., November 15, 1949, Part V.

a 1° C temperature difference is at least equivalent to a pressure of about 100 cm, hydrostatic head. It appears that thermo-osmosis is likely to play a very considerable role under the conditions described by Mr. Wooltorton.

The heat released in wetting of a dry clay which is a representative function of its water affinity normally decreases about 3% for a 10° C increase in temperature. On the other hand, the thermo-osmotic water conductivity of a soil is a very sensitive function of its moisture content having a maximum in the vicinity of the plastic limit. For this reason it is not surprising that the thermo-osmotic coefficients given in Table 6, and the respective hydrostatic equivalents of temperature differentials, are considerably lower than those computed from the well known temperature function of the water affinity.

TABLE 6.—THERMO-OSMOTIC AND ELECTRO-OSMOTIC COEFFICIENTS OF PERMEABILITY

Soils (1)	Thermo-osmotic Experiments		Electro-osmotic Experiments		Volts per centimeter = 1° C per centimeter (Col. 3 divided by Col. 5) (6)
	Average moisture content (%) (2)	Permeability, in centimeters per second per degree centigrade per centimeter (3)	Moisture content (%) (4)	Permeability, in centimeters per second per volt per centimeter (5)	
(a) Pure H-clays (Montmorillonitic, Illitic, and Kaolinitic)					
H-bentonite	61.82	1.91×10^{-6}	....	....	....
H-grundite	26.50	1.61×10^{-6}	36.45	2.86×10^{-6}	0.563
H-kaolinite	30.62	3.39×10^{-6}	30.84	12.8×10^{-6}	0.265
(b) New Jersey Hagerstown Soil					
H-soil	22.53	1.92×10^{-6}	22.60	10.30×10^{-6}	0.186
Al	23.05	1.49×10^{-6}	23.08	7.49×10^{-6}	0.199
Mg	22.70	2.24×10^{-6}	22.57	5.10×10^{-6}	0.439
K	21.85	2.5×10^{-6}	21.83	3.75×10^{-6}	0.667
N	21.46	0.885×10^{-6}	22.29	1.29×10^{-6}	0.686
Ca	22.66	1.63×10^{-6}	24.39	1.06×10^{-6}	1.54
Natural	23.12	3.69×10^{-6}	23.42	2.32×10^{-6}	1.59

Furthermore, it should be realized that nature very seldom operates in accordance with one single mechanism; rather, it uses all available paths (although to different extents) to achieve its purpose. For this reason reference is made[20,21,22] to several papers that contain data and concepts pertinent to the problem of water movement under the soil and climatic conditions with which Mr. Wooltorton's paper is concerned.

[20] "Mechanism of Water Attack on Dry Cohesive Soil Systems," by Hans F. Winterkorn, *Soil Science*, Vol. 54, 1942, pp. 259–273.

[21] "Theoretical Aspects of Water Accumulation in Cohesive Subgrade Soils," by Hans F. Winterkorn and Henry Eyring, *Proceedings*, Highway Research Board, National Research Council, Vol. 25, 1945, pp. 422–434.

[22] "Importance of Volume Relationships in Soil Stabilization," by Hans F. Winterkorn and Dutta Choudhury, *ibid.*, Vol. 29, 1949 (in press).

J. E. JENNINGS.[23]—At a very appropriate time, Mr. Wooltorton has called attention to the behavior of foundations on desiccated clays, since many workers in various parts of the world are facing the same problem. It appears that similar conditions are encountered in the greater part of southern Africa, parts of Australia, Palestine, and also in the high-lying areas of some states adjoining the lower Mississippi River in the United States. The Texans were probably the first to make serious attempts to solve the troubles experienced with buildings on these soils, but it is also very probable that the exact mechanism underlying the movements which these buildings experience has only recently been given detailed consideration.

It is unquestionable that, in South Africa, the movements experienced under such conditions may be broadly described as dome-shaped heaving, with the greatest rise at the centers and the smallest rises at the corners of the buildings. The cracking pattern described by Mr. Wooltorton is quite typical,

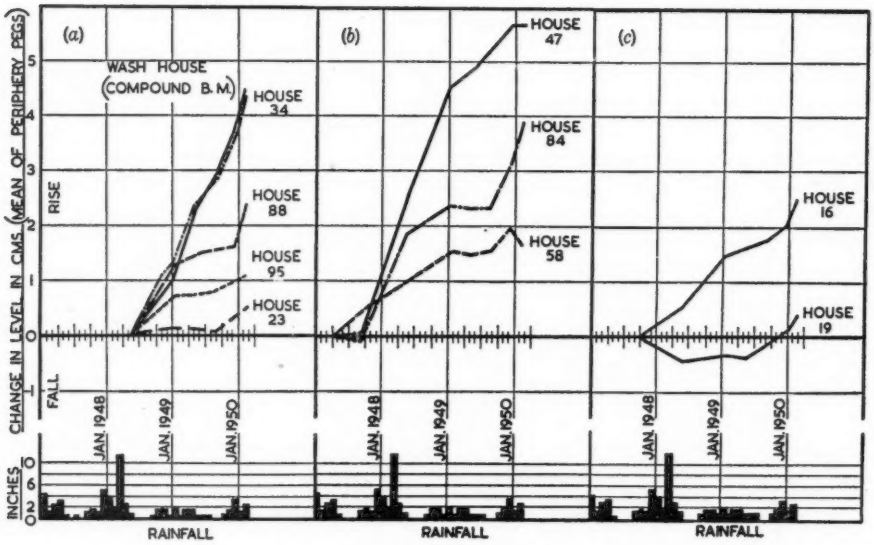

FIG. 17.—HEAVING OF BUILDINGS, ST. HELENA GOLD MINE, ORANGE FREE STATE

and has been classified in South Africa as apparent "corners down"—the same pattern that would be expected where settlement is primarily caused by shear distortion, except that the movement is upward instead of downward. It is important to appreciate that the cracking does not result from settlement of the corners, but from the fact that the corners have risen less than the other parts of the building.

The general cracking pattern described is frequently masked by local irregularities in the soil, and buildings immediately adjacent to each other demonstrate entirely different heaving patterns, as can be noted from Fig. 17, which shows level observations on a group of ten single-story concrete houses, most of

[23] Director, National Bldg. Research Inst., Pretoria, South Africa.

which are within a radius of one-half mile. The pattern is further distorted by local conditions of drainage; and broken drain pipes, or other poor drainage conditions, immediately adjacent to the buildings, have frequently resulted in a dish-shaped rather than a dome-shaped heaving.

Fig. 17 shows actual level observations referred to bench marks established in open fields at depths of approximately 10 ft. Each bench mark consists of a rod, set in concrete at the bottom of a pit. This rod is sleeved with a pipe set clear of the concrete pad, and the hole is then filled back. Check levels between such bench marks have indicated that small movements ($\pm$ 0.2 cm) are taking place, but these are considered to be of second order of magnitude as compared to the movements of the buildings. In future efforts to obtain absolute movements, bench marks in South Africa will be placed in boreholes at least 50 ft to 100 ft deep and provision will also be made for temperature compensation of the rod.

Accepting these bench mark variations as being of second magnitude, it has been observed that pegs driven into the surface of open ground experience upward and downward movement, depending upon rainy or dry weather. With buildings, however, the conditions appear to be different, since they show a general tendency to settle immediately after construction, and then heave progressively for a long time. A correlation between this heaving and the rainfall seems problematical; and, in South Africa, it is considered that the effect of the seasons is only secondary. One series of observations, begun in 1941, indicates that an asymptotic value of the heaving will eventually be reached; and it is probable that, only at this stage, the cyclic movements due to the seasons will have a more important effect. It is also probable that a seasonal cyclic variation is superimposed on the general heaving curve and very accurate, short-period, level observations are being undertaken to ascertain whether this is the case.

In view of the apparent independence of season, other causes for the heaving have been examined and two theories have been propounded. One theory is based on the thermo-osmotic transfer of moisture toward the region of lower temperature in the soil under the building. This region of lower temperature is created in hot climates by the shielding effect of the structure. In this theory the moisture transfer takes place in the capillary fringe and consequently it is a necessary condition that the soils are partly saturated. The other theory relates to the relief of capillary stress, which results from the interference with the surface evaporation following erection of the building. Both theories have been discussed in other papers.[24]

Because of critical variations in the soil and the need to allow for a changing distortion pattern, it is apparent that a solution covering both dish-shaped settlement and dome-shaped heaving must be sought. Three solutions have been proposed in South Africa. The first adopts the principles of the Texas underreamed piles which are used to anchor the building in the zone of constant moisture content. This zone is believed to lie below the free water line in the

[24] "The Theory of Heaving of Buildings on the Shattered Desiccated Clays of Southern Africa," by J. E. Jennings, *First Annual Report*, International Soc. of Soil Mechanics and Foundation Eng., 1950 Section VI*d*.

soil; and, in soils of this type, the free water line is not very easy to recognize, generally requiring a deep excavation that must be examined by a trained observer. The piles are anchored at this level and the building is kept clear of the surface. The second solution is to reinforce the walls of the structure in such a manner that the building can tolerate the movements without cracking. This process is in the development stage and the washhouse in Fig. 18 is a successful example of such practice. The third method makes use of a flexible structure to allow for movements without deleterious or dangerous cracking. Dry-block construction is an example of such a procedure.

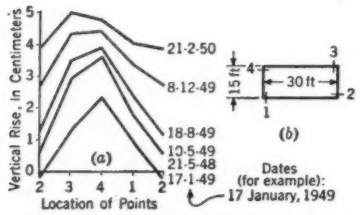

FIG. 18.—DETAILED FOUNDATION MOVEMENTS OF WASHHOUSE (REINFORCED CONCRETE BLOCK ON STRIP FOOTINGS), ST. HELENA GOLD MINE, ORANGE FREE STATE

Mr. Wooltorton has quoted successful practice based on making the foundation pressure approximately equal to the swelling pressure of the soil. It is believed that swelling pressure, measured by the method proposed by L. F. Cooling, will vary, depending on the initial moisture content of the sample. In South Africa swelling pressures are found to be of the order of from 3 tons per sq ft to 5 tons per sq ft, and in one area several successful buildings have been erected using a foundation pressure of approximately 3 tons per sq ft. The method seems open to some doubt, however, for two reasons: (1) The swelling pressure will depend on the moisture content applying at the time of construction; and (2) shearing tests in these soils have indicated that care should be exercised before allowing too high a bearing pressure to be used. Shear tests of soils in South Africa have indicated that bearing pressures should be limited to not more than 1.5 tons per sq ft; but, in view of the success with the aforementioned buildings, the entire testing procedure is being examined. Until this procedure has been developed and fully tested in the field, it is probable that this approach will not find general application in South Africa.

In conclusion, experience in South Africa has shown that the movements vary considerably, even on the same site, and also that the movements experienced by a building are radically different from those on open ground. It is believed that any field observations must be conducted on full-scale structures. Detailed level observations, such as those given in Figs. 17 and 18, have considerably assisted in clarifying a problem that appears fundamentally different from problems experienced in the countries where the principles of soil mechanics have been most generally applied.

F. L. D. WOOLTORTON.[25]—The contributions by Messrs. Winterkorn and Jennings are of particular value in emphasizing the importance of pedological engineering studies in conjunction with soil mechanics. This especially applies to desiccated soils and, more particularly, to such soils under tropical

[25] Constr. Engr., Road Constr. Div., Public Works Dept., Nakuru, Kenya Colony, East Africa.

conditions. As world development gets under way so will the need for, and the importance of, pedological engineering be increasingly felt.

Some of the constructional difficulties experienced in the expansive desiccated clays of Texas together with some of the design methods found successful in that state (including the use of suspended floors and foundation soil ventilation) have been described by Willard E. Simpson,[26] M. ASCE. Incidentally, while Mr. Simpson and the writer were motoring through San Antonio, Tex., they stopped to measure the distance between the crests of a length of waved concrete road curbing and found them to te 62 ft against the value of CE = 67.5 ft in Fig. 11.

Apparently, the following extract from Mr. Simpson's paper[26] agrees with the experience of Mr. Jennings:

> "It has been noticed that the point of maximum swelling is not near the surface, but at a point three to six feet below the surface. The swelling of the soil immediately on the surface is not so forceful but it is equally troublesome."

In the absence of further data, the following possibilities may feature in the explanation:

a. The presence of a higher clay content or of more active clay minerals at this depth (which is not uncommon).

b. The presence of certain residues of organic matter, in the upper 5 ft, acting as protective colloids masking or changing the physioco-chemical properties of the clay fraction.

c. The Texas and South African[24] swelling problem may be aggravated, in some way, by the phenomena of swelling, "softening," and sometimes creep, which characterize some shattered clays after exposure by excavation. These aspects have been discussed by Karl Terzaghi,[27] Hon. M. ASCE, and A. W. Skempton.[28] The two phenomena have similarities but should not be confused. The swelling phenomenon under consideration is not necessarily accompanied by partial softening. It is a result of the release of internal stress and not of a reduction in external loading. It is a property of the clay material modified by the soil structure within its natural elements (which may be infinite in size); whereas the swelling and softening of stiff-fissured clays is a property of the clay modified by the structure of the entire profile rather than that of the individual elements into which the profile is fissured or fractured.

In the former instance, the swelling pressure may be represented as the loading to be applied to a desiccated soil to prevent swelling when in contact with water. It is determinable from the loading curve.

In the latter instance, the swelling or rebound occurs in a clay, already site saturated, as a result of the removal of a known external loading. It is often determined from the appropriate unloading curve.

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[26] "Problems in the Design of Foundations," by Willard E. Simpson, *Refiner and Natural Gasoline Manufacturer*, September, 1935, p. 426.

[27] "Stability of Slopes of Natural Clay," by Karl Terzaghi, *Proceedings*, International Conference on Soil Mechanics and Foundation Eng., Vol. I, Harvard Univ., Cambridge, Mass., June, 1936, p. 161.

[28] "The Principles and Application of Soil Mechanics," by A. W. Skempton, Inst. C. E., London, 1948, p. 51.

Presumably, when the stress is reduced in saturated stiff-fissured clays, water may move comparatively fast through the profile, by way of cleavage planes. The result is to produce some swelling and possibly some slaking of the natural element surfaces. This moisture movement cannot occur with the same rapidity and in such quantities in the nonshattered Burma soils where upward moisture movement must be of the thermo-osmotic type rather than in the capillary state.

It was estimated, in Mandalay (Burma), that no soil movement would occur at a depth of 26.5 ft. This compared with the 25 ft found by Henry C. Porter[29] to apply in parts of Texas. In consequence, no light building constructed on shallow piles could be expected to be generally satisfactory. Some such buildings had been constructed in Mandalay on 9-in. by 9-in. underreamed piles, but where the piles transmitted less than about 10 tons (that is, about 2.5 tons per sq ft) there was cracking in the superstructure.

Successful designs, developed in Mandalay, included both heavy buildings (transmitting loads in excess of 1.5 tons per sq ft) and light flexible buildings, although there was considerable distortion of the latter. Aids to reduce the movements in, and damage to, light flexible buildings included:

1. Tanking the floors and sides of the excavations with a 3-in. layer of selected soil treated with calcium chloride at the rate of 1.5 lb per sq yd and backfilling with selected nonexpansive material;
2. Placing the foundations below the depth of soil weathering in the layer of maximum apparent density (which was also the depth of maximum pH, minimum moisture content change, and minimum rate of moisture change) and always below any lenses of sandy clay loam; and
3. Framing with timber posts bolted to tie beams in the floor and the eaves and to reinforced concrete strip foundation beams.

Mr. Jennings, who recorded that observation in South Africa, suggests that an asymptotic value of the heaving may be reached in about eight years, after which seasonal cyclic movement may be found to occur. The many Mandalay buildings examined partly support this belief. It is obvious that heaving had been incrementally accumulative, but as all the observed buildings were at least ten years old (some of the order of a hundred years) this asymptotic condition might be considered to have been reached prior to observations. Nevertheless there was evidence, for certain buildings, that any condition in equilibrium with the average climate had been frequently disrupted by unusual climatic conditions and a new series of movements initiated.

It would not be expected, from the point of view of pure vertical movement, that an uncovered level ground surface, in equilibrium with the climate, would indicate any appreciable relative movements other than those caused by micro-changes in the topography. These are only to be expected under abnormal conditions of moisture distribution, as when evaporation is hindered over a limited area.

[29] "Observations of the Texas State Highway Department on the Subsequent Effects of the Uniformity and the Non-Uniformity of Foundation Soil-Types on Pavements; and Also the Effects of Uniformity and Non-Uniformity of Moisture Content Fluctuations in Soil Foundations of High Volumetric Change," by Henry C. Porter, *Proceedings*, International Conference on Soil Mechanics and Foundation Eng., Vol. II. Harvard Univ., Cambridge, Mass., June, 1936, p. 256.

Mr. Winterkorn's remarks and data on permeability not only illustrate the care that may be necessary in applying the results of permeability experiments to a design problem but the important role played by pedological processes on the resultant properties of a soil profile.

The writer accepts with gratitude Mr. Winterkorn's correction of "distillation effect" to "thermo-osmotic flow." The phenomenon described by Mr. A. F. Lebedev[6] in 1927 would appear identical with that under discussion but unfortunately he was at a loss for a more suitable descriptive term. To illustrate how engineering pedology has advanced the knowledge and understanding of soil phenomena in the engineering world, and has even led in the agricultural field, it may be added that when the writer discussed the possibility of appreciable moisture flow in the vapor or film phase with authorities in 1936 his data were greeted with considerable doubt. It is now known, thanks to Mr. Winterkorn and the Road Research Laboratory (England),[30,31] that considerable moisture may move through desiccated soils under a temperature gradient, especially when the average moisture content approaches the plastic limit as in Burma. This moisture flow may result in a change in the soil's physical properties; and, when evaporation is hindered, it may create a moisture condition and shearing resistance very different from what might have been expected. In two instances free water was found in the soil material under the floors of buildings examined in Burma during the dry weather.

This phenomenon is of considerable importance to the highway engineer.

Mr. Winterkorn refers to soil swelling pressure as being normally of greater importance to the highway engineer than to the structural engineer. The effect on highway design has been discussed by the writer[32] and investigated by Mr. Winterkorn[22] who suggests that one reason why building engineers have not given more attention to certain soil-physical phenomena may be that the type of buildings adversely affected by such phenomena represent too small a capital outlay to merit detailed studies and protective measures. This attitude, however, should not apply to organizations controlling a large number of such structures. It is noteworthy that the Great Indian Peninsula Railway adopted this policy in 1948.

Is it generally true that soil movements are only detrimental to light structures? Table 7 gives some of the information available. The values of the swelling pressure given should be treated with caution as the conditions under which they were determined are unknown. It may be noted that, in general, the procedure for selecting a suitable foundation bearing pressure is not the same for the A-6 sodium illite-montmorillonite soils of Burma (FME = 18) as it is for the more usual A-7 calcium-bentonite soils of Texas (FME = 35) and that the procedure in South Africa, where the soils appear to be A-7 or A-6 illite-montmorillonite soils (FME = 35 to 20?) is somewhat undecided, sometimes corresponding to the Texas method and sometimes to the Burma method.

Whereas, functionally, the Mandalay soils are structurally homogeneous, the South African and Texas soils are generally shattered.

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[30] "Moisture Movements Occurring in Soil Due to the Existence of a Temperature Gradient," Road Research Laboratory (England), *Note RN* 761 *DJM*, May, 1946.

[31] "Soil Moisture and the Factors Governing Its Movement," Road Research Laboratory (England), *Note RN 944 DC*, February, 1948.

[32] "Relation Between the Plastic Index and the Percentage of Fines in Granular Soil Stabilization," by F. L. D. Wooltorton, *Proceedings*, Highway Research Board, National Research Council, Vol. 27, 1947, p. 479

It appears reasonable that, in principle, the design pressure for light buildings, constructed on desiccated swelling soils, should be sufficient to overcome the effective swelling pressure if differential wall movements are to be avoided; and when this is not possible, because of a limiting soil shear resistance or cost considerations, the design must provide for distortion stresses. Massive buildings—other than their ground or basement floors—may possibly be able to resist distortion effects through their inertia even though the pressure transmitted is less than the effective swelling pressure.

TABLE 7.—Relative Effect of Soil Movements on Light and Heavy Structures

Location	Clay minerals	Desiccation (uncovered state)	Swelling pressure[a]	Clay shattered	Field moisture equivalent	Type of building affected
(1)	(2)	(3)	(4)	(5)	(6)	(7)
San Antonio, Tex.	Calcium bentonitic	Yes; to 17 ft	4 to 5	Yes	35	Light
Cisco, Tex.....	Generally bentonitic	Yes	0 to 7	Yes ?	20	Light to medium heavy (?)
London, England blue clay.....	Illitic (?)	Occasionally, to about 5 ft during droughts	1.2	Yes[b]	30 (?)	Light
South Africa.....	Illite[c]	Yes; to at least 10 ft	3 to 5	Yes	20 to 30 (?)	Light
Mandalay, Burma	Illite[c]	Yes; to at least 12 ft	1.75	No	18	Light to medium heavy

[a] Conditions under which the swelling pressures (in tons per square foot) in Col. 4 were determined are not generally known. Remarks pertinent to each item in Col. 4 are as follows:
San Antonio, Tex.: Suitable bearing pressure was controlled by the low shear resistance of the soil when it was saturated.
Cisco, Tex.: Buildings transmitting pressures of 3.5 tons per sq ft have been affected by soil swelling.
London, England: The swelling pressure was 1.2 tons per sq ft under normal conditions. The design was based on the shearing resistance of the soil.
South Africa: Apparently, the safe bearing pressure was generally determined by shear considerations. Some satisfactorily designed buildings transmit pressures of the same order as the swelling presures.
Mandalay, Burma: Under extreme site conditions, the swelling pressure was 1.75 tons per sq ft. Buildings that transmitted as much as 1.5 tons per sq ft were affected by soil swelling. Since the shear resistance is high, suitable design pressures are limited to a minimum value equal to the swelling pressure.

[b] The clay was fissured or shattered. [c] Illite, with small quantities of montmorillonite.

Where the nature of the clay minerals is important in influencing the maximum value of the swelling pressure, the amount of desiccation, at the time of construction and possibly during periods after construction, will determine the effective swelling pressure and is thus, in effect, of greater importance.

The selection of a suitable design pressure, and frequently of the design type, would therefore appear to involve in its general solution a consideration of two factors:

a. A maximum value based on shear considerations involving a forecast of the maximum moisture content by capillary or thermo-osmotic flow, or both, toward the foundations. For predictory work, this value may perhaps be safely taken as the field moisture equivalent (FME).

b. A minimum value determined by the effective swelling pressure involving a forecast of the minimum moisture content, or provision for resisting distortion stresses. This minimum moisture content would appear to be (in the worst cases of desiccation, although not generally) about equal to the shrinkage limit.

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